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SOIL AND GEOLOGIC FEATURES OF THE BUFORD PROJECT

by William V. Conn

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SOIL AND GEOLOGIC FEATURES OF THE BUFORD PROJECT

William V. Conn*

INTRODUCTION

Buford Dam is located in Gwinnett and Forsyth Counties on the Chattahoochee River about halfway between the towns of Buford and Cumming, in north-central Georgia. It is one unit in the authorized development of the Apalachicola, Chattahoochee, and Flint Rivers. The other authorized units are the Fort Gaines multiple purpose project; the Columbia Lock and Dam; the Jim Woodruff multiple purpose project; and open channel improvement of the Apalachicola River below Jim Woodruff Dam. Buford's functions in connection with these other downstream projects, are: (1) the regulation of low water flow to increase depths and provide lockage in the lower Chattahoochee and Apalachicola Rivers; (2) to raise the primary power capacity at Fort Gaines and Jim Woodruff; (3) operation of the power units at overload during flood periods on the lower river when capacities at Fort Gaines and Jim Woodruff are reduced by high tailwater, thereby firming, or making dependable, capacities at these plants that could not be considered dependable unless so firmed either by Buford or some other plant. There are also several privately owned hydro-electric plants downstream that will be benefited through the increased minimum regulated flow provided by Buford.

Topographically, and from a foundation standpoint, the site appears ideally suited for a conventional concrete dam, but cost studies indicated that a main dam of earth fill with an abutment type powerhouse would be the most economical scheme of development, and the earth structure was, therefore, adopted. The general layout of the project is shown on Figure 1. Deep cuts have been made in the right abutment for the powerhouse and intake structure, and the rock from the excavation will be utilized to provide a heavy rock section at the upstream face of the main earth dam. Approximately 1,000,000 cubic yards of rock were excavated from these forebay and tailrace cuts, 800,000 cubic yards of which will be used in the main earth dam, which will make the structure, strictly speaking, a modified earth and rock fill design.

Three tunnels through the rock of the right abutment will connect the reservoir with the powerhouse. Two of the tunnels, with a diameter of 22 feet after lining, serve as penstocks to deliver the water to the two 40,000 KW main units. A third auxiliary unit, with a capacity of 6,000 KW, will provide necessary downstream flow during off-peak periods when the two main units are not operating. The third tunnel, with a lined diameter of 13.25 feet, will be utilized for diversion during construction, and will be used thereafter to

* Engineering Geologist, Corps of Engineers, U. S. Army, South Atlantic Division, Atlanta, Georgia.

evacuate flood waters from the reservoir only for floods greater than any of record or for other floods should the power units be inoperative.

To provide spillway capacity for the theoretical maximum flood - which is a flood stage greater than ever recorded previously, and one which may never actually be attained - an unpaved chute spillway was excavated through the reservoir rim approximately one mile east of the main structure. The earth and rock excavated to form this cut was utilized for the construction of a saddle dike required by a low saddle in the reservoir rim adjacent to the spillway.

Construction at the project to date has involved completion of the spillway and three saddle dikes, and excavation of the forebay, tunnels, and tailrace. Lining of the tunnels and construction of the headworks is underway, and a contract has recently been let for the construction of the main earth dam. The final phase of the work, construction of the powerhouse, is expected to begin in March, 1956, and the first power is scheduled to be produced in December, 1957.

Geology and Soils of Project Area

The predominate rock underlying the earth dam and appurtenant structures is a steeply dipping biotite granite gneiss belonging to the Carolina series of pre-Cambrian age. At depth, where fresh and unweathered, it is an extremely hard, durable, and impermeable rock, and provides an excellent foundation material. At shallow depths, however, where it has been exposed to the agents of weathering, the feldspars and micas have broken down developing zones of weakness along the dipping gneissic structure. These zones, complicated by continuous vertical joint planes developed along the strike, have combined to produce serious excavation problems in the deep cuts of the forebay and tailrace areas.

The soils underlying the dam in the valley floor are alluvial materials, chiefly micaceous sands and silts. They are of limited depth and extent with respect to the design of the dam. The soils of the valley slopes and borrow areas, from which the structure will be constructed, are residual and were derived from the weathering of the granite gneiss parent rock. They range from a clayey sand containing appreciable quantities of mica to a highly micaceous silty sand which grades gradually into weathered rock and exhibits more and more of the rock structure as it approaches the top of firm and unweathered granite gneiss. On the basis of an extensive boring program a borrow area was selected extending over 600 acres and estimated to contain 10,000,000 cubic yards of available material. Throughout the borrow area the upper portion of the material, comprising 15 to 20% of the available borrow, exhibits a plasticity index of 6 to as high as 20. Usually this upper plastic soil is red, while the underlying non-plastic material is gray. A controlled blend of these two soil types will be utilized in construction of the earth fill section of the main dam.

General Features of Earth Dam

The design of the main dam follows conventional earth dam construction procedures, the important features of which are indicated on Figures 2 and 3. The use of a relatively large upstream rock zone is made possible by utilizing 800,000 cubic yards of rock excavation secured from the construction of the forebay, tunnels, and tailrace. The downstream slope is unbroken by berms

except for a low roadway berm to provide access to the powerhouse. A conventional cut-off trench will be excavated to firm rock, and foundation percolation will be further minimized by the construction of a grout curtain along the dam axis. Although the granite gneiss, where sound and unjointed, is highly impermeable, the grout curtain is considered necessary to insure an adequate cut-off in jointed areas. A downstream gravel drainage blanket will collect embankment seepage and insure that the saturation line will not intersect the downstream face. The blanket will extend into the downstream rock toe.

The embankment will be provided with a central, relatively impermeable zone of selected borrow material compacted to high density to insure a low permeability core for the earth structure.

An interesting feature of the project, and one which greatly influenced the earth dam design, was the inclusion of a test fill area in the construction of saddle dike No. 3. This dike, requiring approximately 500,000 cubic yards of fill, was the first portion of the project to be constructed, and it provided an ideal opportunity to experiment with compaction procedures so that the most effective and economical methods of placement and compaction could be developed for use during the subsequent construction of the main earth dam. The general layout of the test sections is shown on Figure 4.

The two predominant soil types encountered in the borrow area and studied in the test fill have been designated, for convenience, as soil "A" and soil "B". The red, plastic, surface soils are referred to as soil "A", and the gray, non-plastic, silty sands are called soil "B".

The test fill was divided into two separate areas to permit easy integration into the construction schedule. Test track No. 1 was 100 feet wide by 375 feet long and was divided into 20 x 40-foot subdivisions or areas, each of which would be subjected to varying amounts of moisture and varying types and amounts of compaction. Test track No. 2 consisted of only two sections, each 20 feet by 50 feet.

Earth fill material for test tract No. 1 was selected so that sections 1 through 5 used soil "A" and sections 6 through 10 used soil "B". All layers of the test track were spread 9 inches loose or 6 inches compacted thickness. Efforts were made to vary moisture content from 4 percent above optimum to 4 percent below optimum in each of the sub-sections. Each of the sub-sections was also subjected to varying compactive efforts by both a 78,000 lb., 4-wheeled rubber tired roller and a modified 69 inch, double drum, sheepsfoot roller. Two, four, and eight passes of the rubber tired roller, and 6 and 12 passes of the modified sheepsfoot roller were used to provide the compactive effort. Thus the test track was subjected to controlled variation in soil classification; type of roller; number of passes; and attempted variation in moisture content.

Test track No. 2 was compacted with soil type "B" only. The track was 40 feet by 50 feet and was divided into two sections, each 20 feet by 50 feet. The soil was spread 18-inches deep to secure data on the effectiveness of rollers in compacting thick layers.

A study of the action of the standard sheepsfoot roller being used for compaction of the saddle dike indicated that the weighted standard roller continually rode on the drum, reducing the effective compactive effort on the tamper feet. An experimental modification to a standard roller was made by welding 3 by 4-sq. in. steel plates to each tamping foot to increase its area to 12 sq. in. from the usual 5 to 7 sq. inches. This modification was successful in making the roller "walk out" and will be referred to in the subsequent discussion as the "modified roller."

During construction of the test fills frequent density and moisture content tests were made, and at the completion of each test track continuous undisturbed samples were taken of each section for laboratory testing.

Upon completion of both the field portion of the test fill and the laboratory testing of the samples, the extensive data which had been secured was compiled and carefully analyzed to provide the best possible information on the soil characteristics, and thus be of maximum benefit for use in the design of the main earth dam.

The data obtained from the test fills indicated that no soil characteristics would be encountered which would present serious construction problems. However, some of the information disclosed rather unusual or unanticipated fill characteristics, and these are summarized as follows:

1. Moisture control on both "A" and "B" type soils will require close supervision. Soil "A", as it came from the borrow pit, was about 4% above optimum moisture. Efforts to appreciably decrease the moisture content failed because of high humidity, frequent showers, and condensation and absorption of dew during the night. Thus it proved extremely difficult to secure the desired moisture, and later laboratory determinations showed a nearly constant moisture content, above optimum, for the soil "A" subsections.

2. Similar difficulties, except in reverse, were experienced with soil "B". The material came from the borrow pit about 4% below optimum. Efforts to increase the moisture to optimum and above were only partially successful. Apparently the non-plastic and granular characteristics of the material provided sufficient drainage to make it difficult to obtain moisture at optimum for any length of time. However, it was found that moisture content was not critical with respect to the compaction qualities of the material, at least so long as the moisture reached a certain minimum, approximately 6% below optimum. Additional improvement in density was also obtained by increasing the number of roller passes.

3. Results of the experimental compaction tests showed that at the moisture contents used there was little preference in the method of compaction to be selected. Additional passes of the modified sheepsfoot roller added some benefit to the compaction, but additional passes of the rubber tired roller showed virtually no advantage. On test track No. 2, where 18-inch layers of soil "B" were compacted, it indicated that compaction of thicker layers would be practicable with the rubber tired roller. Both modified sheepsfoot and rubber tired rollers resulted in better compaction for both soils "A" and "B" than the standard sheepsfoot roller which had been specified and was generally used in the construction of saddle dike No. 3.

Influence of Soil and Geologic Features on Construction

During the construction which has been accomplished to date at Buford Dam, two characteristics of the foundation material have caused construction problems unique to the soils of the project. The solution of these problems is considered sufficiently important to be described in detail, as similar conditions might be encountered in any large excavation project conducted in residual soils and igneous or metamorphic rocks.

The basis of both problems is the characteristic manner of weathering of the granite gneiss into the residual soils. The resulting material varies from hard, fresh rock through all stages of rock decomposition to the end product represented by the red sandy micaceous clays which form the surface soils. Contract specifications for the excavation of the spillway provided separate

pay items for common and rock excavation. In practice, however, it was extremely difficult to distinguish between common and rock. The so-called common excavation contained many residual lenses of hard rock, sometimes extending over a large portion of the excavation, which required blasting for removal. Underlying this rock, however, was often a considerable thickness of unquestioned common, into which the blasting of the rock lenses mixed the rock fragments, making determination of the amount of rock to be paid for practically impossible. To remedy this situation during the subsequent contract for excavation of the forebay and tailrace, a simple and effective solution was decided upon. Based on the unusually complete boring information available, a line was selected which represented an arbitrary classification division. All material above the line was considered unclassified, and all the material below the line was considered rock, regardless of the type of material encountered. The workability and convenience of this payment procedure has been demonstrated during the recently completed contract for the excavation of the forebay and tailrace, which involved payment for the removal of 770,000 cubic yards of common and 900,000 cubic yards of rock. At no time was there any question between the contractor and the government as to the classification of the excavated material, which was not only an aid toward more cordial contractural relations, but which actually permitted a considerable government saving in survey costs.

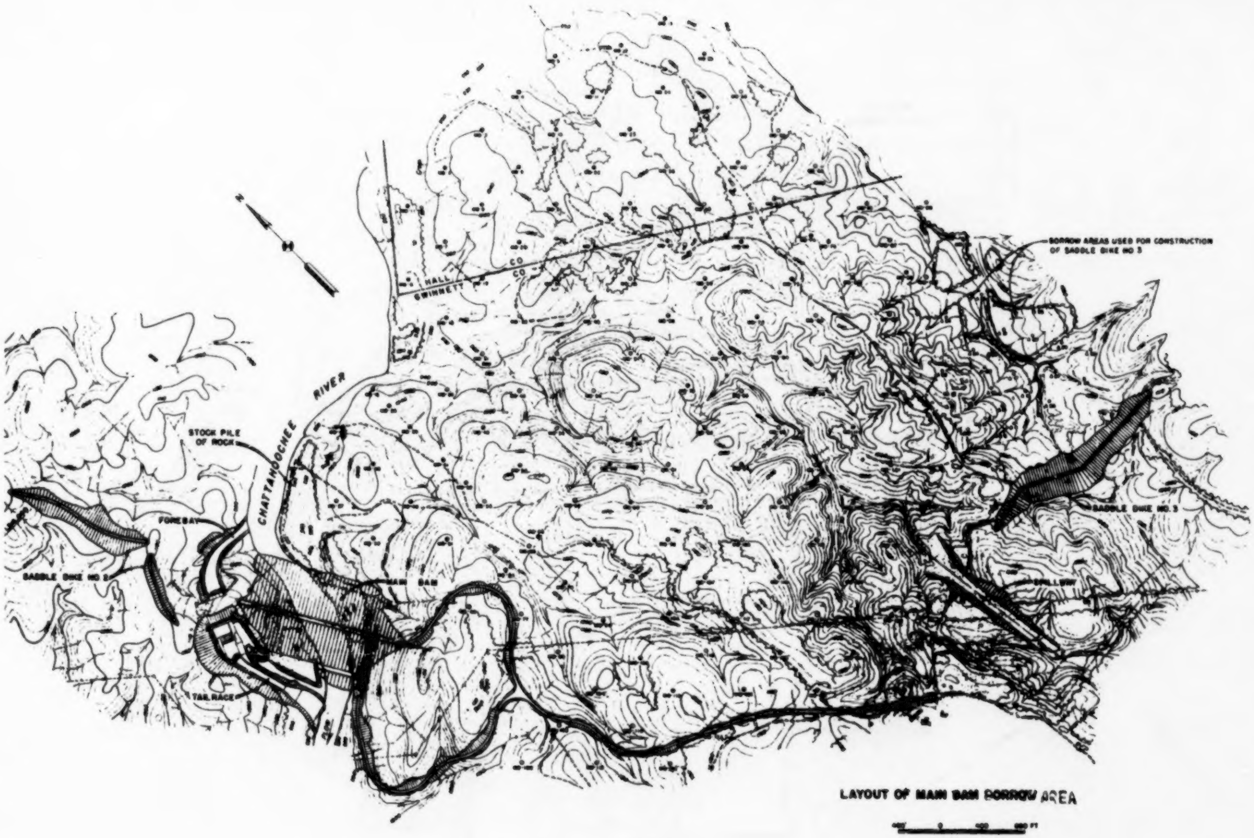
The second characteristic of the foundation material which greatly effected construction procedures was the ability of the residual soils to retain the structure of the parent rock, even though rock decomposition had progressed to the point that the material was unquestionably a soil and bore little physical resemblance to the original rock from which it was derived. The original rock contained a complex joint system, including strike, dip, and oblique joints. These were further complicated by planes of weakness along the gneissic structure usually caused by highly foliated concentrations of biotite. When the rock containing these planes of weakness altered into a residual soil, the planes became an inherent part of the resulting soil. Although the forebay and tailrace areas were adequately explored by borings, these residual planes were extremely difficult to detect and trace, as the only way they could be outlined with certainty would be by exploration with a grid of continuous undisturbed samples - a prohibitively expensive procedure. They may be detected by rock borings only at depths great enough to provide complete core recovery. This expedient, however, is not reliable, as any attempts to project rock joints into the overlying soil by strike and dip projection is at best an approximation. The planes of soil weakness thus present a difficult problem in slope design, since the conventional procedures of soil mechanics in determining slope stability can be used only if the extent, direction, and shear characteristics of the joint planes are known. This was evidenced by a slide which occurred in the tailrace section of the Buford Project. Adequate slope stability for soil overlying the tailrace excavation had been determined to be 1 vertical on 2 horizontal, and excavation proceeded on that slope. After approximately 100 feet of overburden had been removed, and weathered rock was approached, the slope failed along residual planes of weakness parallel to the gneissic structure, aggravated by residual vertical joints and the removal of support by excavation of the slope. The fact that the gneissic structure at this point dips toward the excavation was also unquestionably a contributing factor in controlling the direction of the slide. Remedial measures to completely stabilize the slope required removal of approximately 88,000 cubic yards of overburden and weathered rock.

Occurrence of similar slides should be recognized as being possible in any excavation in residual soil when the optimum condition of residual jointing and alignment of the rock structure exists. Anticipation of such slides is difficult, but their occurrence may be minimized by:

(1) Adequate borings to determine the direction of rock dip in relation to the excavation:

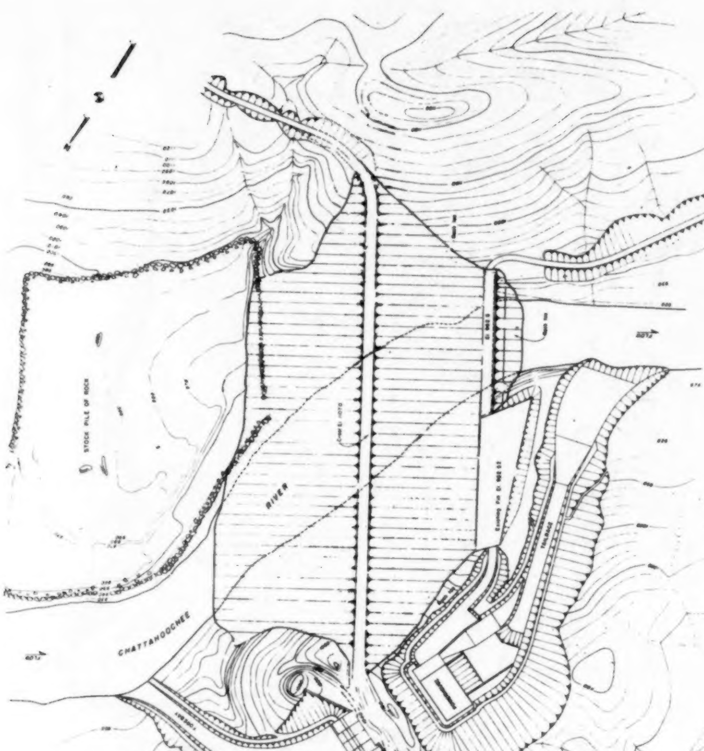
(2) A close study of the entire soil profile as disclosed by splitspoon and undisturbed borings:

(3) Utilization of an unusually large safety factor for slope stability design in areas of jointed residual soils.



BORROW AREAS USED FOR CONSTRUCTION OF SADDLE DIKE NO 3
HALL CO
GWINNETT CO
STOCK PILE OF ROCK
CHATTAHOOCHEE RIVER
FOREBAY
SADDLE DIKE NO 2
MAIN DAM
TAILRACE
SADDLE DIKE NO 3
SPILLWAY
LAYOUT OF MAIN DAM BORROW AREA

Figure 1



GENERAL LAYOUT
MAIN DAM

Figure 2

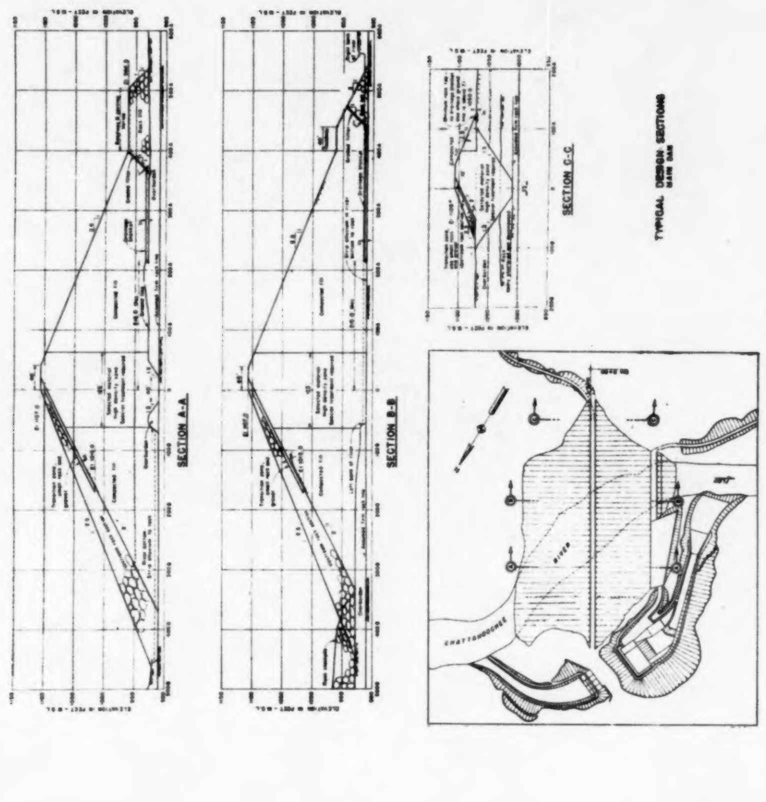

SECTION A-A
SECTION B-B
SECTION C-C
TYPICAL DESIGN SECTIONS
MAIN DAM
CHATTAHOOCHEE
RIVER

Figure 3

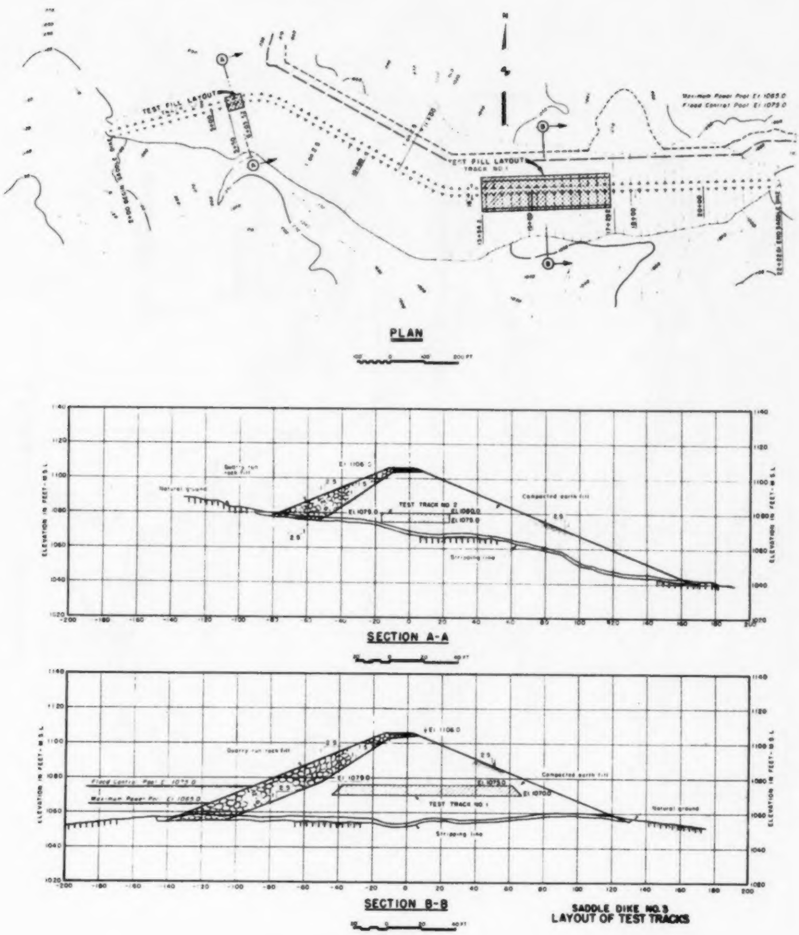

TEST FILL LAYOUT
TEST FILL LAYOUT
TRACK NO. 1
PLAN
Natural ground
Compacted earth fill
TEST TRACK NO. 2
Stripping line
SECTION A-A
Quarry run rock fill
Compacted earth fill
TEST TRACK NO. 1
Natural ground
Stripping line
SECTION B-B
SADDLE DIKE NO.3
LAYOUT OF TEST TRACKS

Figure 4